

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS
PROPOSED CULVERT REPLACEMENT (C5) AT
STATION 11+217, HIGHWAY 522 REHABILITATION,
FROM 32.2 KM WEST OF HIGHWAY 524
EASTERLY 6 KM
G.W.P. 480-98-00, DISTRICT 54, SUDBURY
GEOCRES NO. 31E-279**

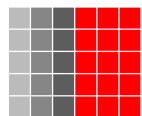
Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

**SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.**

**Project: SPT1218A
November 27, 2008**



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DRAWINGS

DRAWING No.

BOREHOLE LOCATION PLAN & SOIL STRATA

1

APPENDICES

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**FOUNDATION INVESTIGATION REPORT
PROPOSED CULVERT REPLACEMENT (C5) AT
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1. INTRODUCTION

Shaheen & Peaker (S&P), A Division of Coffey Geotechnics Inc., was retained by D.M. Wills Associates to carry out a foundation investigation at the site for a proposed replacement of the existing culvert (Culvert 1 at Station 11+217) under Highway 522 near Port Loring, Ontario. This site is located about 0.9 km north of the junction of Highway 522 with Wilson Lake Crescent in Port Loring, Ontario.

The purpose of the investigation was to obtain information about the subsurface conditions at the site by means of boreholes, and to determine the engineering characteristics of the subsurface soils by means of field and laboratory tests.

The findings of the investigation are presented in this report.

2. SITE DESCRIPTION AND PHYSIOGRAPHY

Port Loring is located about 60 km west of Trout Creek which is located at the junction of Highway 522 and Highway 11. The topography near the site is of a rolling nature, with occasional knobs resulting from bedrock outcrops.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, the site is located within the Physiographic Region known as the Algonquin Highlands. The Quaternary deposits found in this area are quite complex, having resulted from a variety of geological process associated with glacial, glaciofluvial, and glaciolacustrine conditions. A large proportion of the area consists of bare bedrock with thin drift. Much of this region is underlain by Precambrian rocks of the Grenville structural Province. These rocks have been strongly metamorphosed, folded, and then intruded by igneous rock.

According to Bedrock Geology of Ontario Map 2544, the bedrock underlying the site consists of Mesoproterozoic Precambrian rocks (i.e. approximately 900 million years old), primarily felsic igneous tonalite, granodiorite, monzonite, granite, syenite and derived gneisses.

3. FIELD AND LABORATORY WORK

The fieldwork for this project was performed between June 18 and June 24, 2008 and consisted of drilling and sampling three boreholes to depths ranging from 3.0 to 12.7 m below the ground surface. The locations of the boreholes at the site are given on the Borehole Location Plan, Drawing No. 1.

The boreholes were advanced using a track-mounted drilling rig owned and operated by Landcore Drilling of Chelmsford, Ontario under the full-time supervision of technical personnel from S&P. The boreholes were advanced using continuous flight hollow-stem augers. The boreholes were extended by augering to depths ranging from 3.0 to 9.5 m below the ground surface, to refusal depths on the augers. Within these depths, the sampling was effected at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. The test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm O.D. split barrel (SS – split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value of the soil which is indicative of the compactness condition of granular (cohesionless) soils (gravels, sands and coarse silts) or the consistency of cohesive soils (clays and clayey silts).

In Boreholes C1-1 and C1-2, after encountering practical refusal on the augers, the bedrock was proven by diamond drilling and NQ size rock cores were obtained. The lengths of the coring were about 3 m.

Where the consistency of the soil permitted in the cohesive deposits, the undrained shear strength of the soil was measured in-situ by means of MTO field vane tests. As well, relatively undisturbed sample was taken from Borehole C1-2 by means of thin-walled Shelby tube sampler.

Dynamic Cone Penetration Tests (DCPT) were performed from the ground surface adjacent to all the boreholes. In this test, a 51 mm diameter, 60-degree apex cone, screw attached to the tip of an A-size rod, is driven into the ground, using the same driving energy as the SPT method. By recording the number of blows of the hammer to drive the cone/rod assembly, into the soil every 0.3 m, a qualitative record of soil compactness condition is obtained. Although the interpretation of the test results is difficult because no samples are obtained by the DCPT and the penetration resistances are not necessarily equal to the N-values, useful information is gained by the continuity of the results and by the elimination of unbalanced hydrostatic force effects which in some cases (such as the present case) affect the SPT results.

The borehole locations were established in the field by S&P engineering staff, in relation to the existing features. The borehole geodetic elevations were provided by D.M. Wills Associates.

Water level observations in the open boreholes were made during drilling and at completion. In addition, a piezometer was installed at Borehole C1-3 and water level observations in the piezometer were made by subsequent site visits.

Upon their completion, the boreholes were grouted using quick grout slurry.

The soil samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations, grain size analyses and Atterberg Limits tests, was performed on selected representative samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets (Appendix A) and also in Appendix B.

4. SUMMARIZED SUBSURFACE CONDITIONS

Boreholes C1-1 and C1-3, which were drilled at the toe of the embankment, contacted topsoil and organic rich soils to a depth of about 0.5 m, underlain by silty clay to depths ranging from 0.7 to 2.2 m below o.g. Below silty clay, these boreholes show the presence of silt to depths of 2.0 to 3.7 m and this silt layer is further underlain by sand (Borehole C1-1) and gravelly sand (Borehole C1-3). Borehole C1-3 was terminated in the gravelly sand layer at 3.0 m below the o.g. level or at El. 232.7 m while in Borehole C1-1 auger refusal was encountered at 5.3 m (El. 299.3 m) on bedrock and the borehole was advanced 3.1 m into the bedrock by NQ coring and terminated at a depth of 8.4 m.

Borehole C1-2 drilled from the existing road pavement contacted about 5.0 m of embankment fill which is underlain by a 0.3 m thick organic silty clay layer. Below the organic soil and silty clay, Borehole C1-2 shows the presence of silty clay, which is in turn underlain by silty fine sand and fine sand with silt to a depth of 9.5 m (El. 229.2 m). Borehole C1-2 was advanced by NQ coring about 3.2 m into the bedrock and terminated at a depth of 12.7 m.

Details of the stratigraphy encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. The following paragraphs are only meant to complement and amplify these data.

4.1 TOPSOIL / ORGANIC RICH SOIL

In the boreholes drilled from the o.g. level, a 0.1 m thick topsoil layer was encountered in Borehole C1-3. In this borehole, the upper 0.4 m of the underlying silty clay deposit was found to be organic stained. In Borehole C1-1 which was drilled on the west side, the upper 0.5 m of the soil (i.e. silty clay) was found to be organic rich.

In Borehole C1-2 which was put down from the surface of the embankment, a 0.3 m thick organic silty clay layer was found extending to a depth of 5.3 m or El. 233.4 m.

4.2 FILL

Fill materials were contacted in Borehole C1-2 (drilled from the top of the road embankment) to a depth/elevation of 5.0 m / 233.7 m.

Below a 25 mm thick asphalt layer, granular pavement fill (i.e. gravelly sand) was contacted to a depth of 0.7 m. Based on N-value of 11 blows/0.3 m, the relative density of this pavement fill is described as compact.

Underlying the pavement fill, Borehole C1-2 contacted embankment fill materials. These materials consist of sand to sandy silt, sand with some gravel and fine sand. The presence of occasional silt, clayey silt and topsoil pockets was also noted.

The grain-size distribution of two samples from the fill material is given in Figures B-1 and B-2 (Appendix B).

The following grain-size distribution is indicated.

Gravel:	1 - 10 %
Sand:	45 - 72 %
Silt:	20 - 30 %
Clay:	7 - 15 %

Standard Penetration tests performed in the fill yielded N-values which range from 1 to 24 blows/0.3 m. From the recorded N-values, the relative density of these granular fill materials is described as very loose to loose at the bottom 1.3 m of fill, while the rest of the fill materials show generally compact condition at the upper portion. These results indicate that the fill materials above the culvert obvert elevation (i.e. above about El. 235.0 m) have received a systematic compaction when they were first was placed, while in the lower portion (i.e. below culvert obvert elevation) the fill received little or no compaction.

4.3 ORGANIC SILTY CLAY

As mentioned before, beneath the embankment fill, a 0.3 m thick organic silty clay layer was encountered (i.e. organics do not appear to have been properly stripped at the location of Borehole C1-2, prior to placing the embankment fill). N-value recorded in the deposit is 1 blow/0.3 m. This indicates very soft consistency of this clayey (cohesive) soil.

4.4 SILTY CLAY

Underlying the fill and organic soil (Borehole C1-2) and the surficial organic soils (Boreholes C1-1 and C1-3), the boreholes contacted an about 0.6 to 1.7 m thick layer of silty clay.

Laboratory tests performed on four samples in this cohesive deposit yielded the following index values (see Figure B-3 in Appendix B).

Liquid Limit:	34 – 46 %
Plastic Limit:	21 – 27 %
Plasticity Index:	12 - 19 %

These results are characteristic of clayey soils of medium plasticity. From the fact that the measured natural moisture contents are generally in between the measured plastic and liquid limits, the deposit may be somewhat pre-consolidated, possibly due to desiccation.

N-values recorded in the deposit range from 2 to 13 blows/0.3 m. The recorded N-values of 2 to 6 blows/0.3 m in Boreholes C1-1 and C1-3 indicate a soft to firm consistency at the toe area of the embankment. Recorded N-values of 2 and 13 blows/0.3 m in Borehole C1-2 indicate a stiff consistency at the top 0.7 m of the layer and very soft below. Field vane test was performed near the bottom of the layer in Borehole C1-2 and this test yielded an in-situ undrained shear strength of 40 kPa. This shear strength value indicates firm consistency of the clayey soil.

4.5 SILT

Beneath the silty clay, Boreholes C1-1 and C1-3 contacted a 1.5 m and 1.3 m thick silt deposit at depths/elevations of 2.2 / 232.4 m and 0.7 / 235.0 m, respectively.

This is a basically fine-grained granular material. The grain-size distribution of a sample from the deposit is shown in Figure B-4 in Appendix B. The following grain-size distribution is indicated.

Gravel:	6 %
Sand:	22 %
Silt:	56 %
Clay:	16 %

It should be noted that some silty clay and sandy silt interbeds / zones were observed in this deposit.

Standard Penetration tests performed in the deposit yielded N-values which ranged from 6 to 16 blows/0.3 m. These test results indicate loose to compact condition but generally compact.

4.6 BASAL GRANULAR DEPOSITS

Basal granular deposits consisting of silty fine sand, sand and gravelly sand were encountered beneath the silt deposit (Boreholes C1-1 and C1-3) and silty clay (Borehole C1-2).

The grain-size distribution of a sample from the sand and silty fine sand from Boreholes C1-1 and C1-2 are shown in Figure B-5 in Appendix B while Figure B-6, shows the grain-size distribution of a sample from the gravelly sand from Borehole C1-3. The following grain-size distribution is indicated.

Gravel:	7 - 29 %
Sand:	59 – 85 %
Silt&Clay	7 – 28 %

Standard Penetration tests performed in these basal granular deposits yielded N-values which ranged from 11 to in excess of 100 blows/0.2 m. These test results indicate compact condition in Borehole C1-1 and a compact to very dense condition in Borehole C1-2. Recorded N-values recorded in the gravelly sand in Borehole C1-3 indicate a compact to dense relative density.

4.7 BEDROCK

Based on the behaviour of the augers during the drilling process and the samples recovered from SPT, as well as the coring results (where coring was carried out), the surface of the bedrock in Borehole C1-1 and C1-2 was inferred/encountered at the following depths/elevations.

Table 4.6.1
Inferred Bedrock Surface

Borehole No.	Existing Ground Surface Elevation (m)	Inferred Bedrock Surface Below Existing Ground Surface (m)	Inferred Bedrock Surface Elevation (m)	Coring Carried Out
C1-1	234.6	5.3	229.3	Yes
C1-2	238.7	9.5	229.2	Yes

In Borehole C1-3, refusal to further penetration of the split-spoon sampler and to further advancing the augers were encountered at a depth of 3.0 m below the o.g. level or at Elevation 232.7 m. As well, a DCPT was performed adjacent to the borehole and this test encountered refusal at the same elevation. This may possibly represent the surface of the bedrock or a coarse granular soil with frequent cobbles and boulders. It should be pointed out that in Boreholes C1-1 and C1-2, the surface of the bedrock was contacted at similar elevations of 229.3 m and 229.2 m, respectively while the refusal elevation in Borehole C1-3 is 232.7 m (i.e. about 3.5 m higher). From this, it can be surmised that at the location of Borehole C1-3, the surface of the bedrock dips sharply towards Borehole C1-2 and then becomes flat or the refusal depths at Borehole C1-3 do not represent the surface of the bedrock but rather a bouldery layer. The latter appears to be a more plausible scenario.

From the rock cores, the bedrock was identified as a grey coloured gneiss. From a visual and tactile examination of the cores, the rock appeared to be intact with some fractures mostly in the horizontal and sub-vertical directions. The photographs of the rock cores are given in Appendix D. Vertical fracture zones in the rock were noted in the bottom 0.9 m zone of the cores recovered from Borehole C-1 (see Record of Borehole C1-1 in Appendix A and the photograph of the Cores from Borehole C1-1).

The following table presents the percentage of recovery and R.Q.D. values measured on the rock cores obtained in Boreholes C1-1 and C1-2.

Table 4.6.2
Rock Core Information

Borehole No.	Rock Core No.	Percentage of Recovery (%)	Measured R.Q.D.* Value (%)
C1-1	8	96	59
	9	95	33
C1-2	13	91	76
	14	100	84
	15	100	85

*R.Q.D. = Rock Quality Designation is the sum of those intact core pieces, 100 mm+ in length, expressed as a percentage of the total length of coring run.

From the above table, it is noted that the total recovery (i.e. percentage of recovery) of the rock cores from the bedrock ranges between 91 and 100 %. R.Q.D. values range from 33 to 85%. Based on these results, the rock mass quality is described as poor to good.

4.8 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed during the drilling and at the completion of each borehole. The observations are shown on the individual Record of Borehole sheets.

A piezometer was installed in Boreholes C1-3 at a depth of 2.1 m. The water levels encountered in the piezometer upon completion and after several days of installation were 0.15 and 0.2 m below ground surface.

Based on the information, the groundwater table at the time of our investigation was at about o.g. elevation.

It should, however, be pointed out that the groundwater at the site would be subject to seasonal fluctuations as well as fluctuations due to weather events and the water level in the water course.

SHAHEEN & PEAKER



Gwangha Roh, Ph.D.



Ramon Miranda, P. Eng.



Zuhtu S. Ozden, P.Eng.



ZO:tr/idrive

Drawings



METRIC

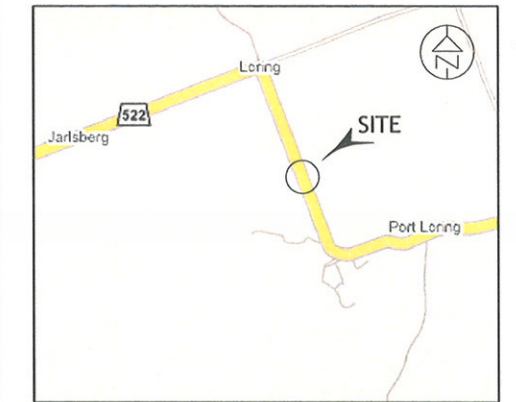
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.

NOTES:
FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

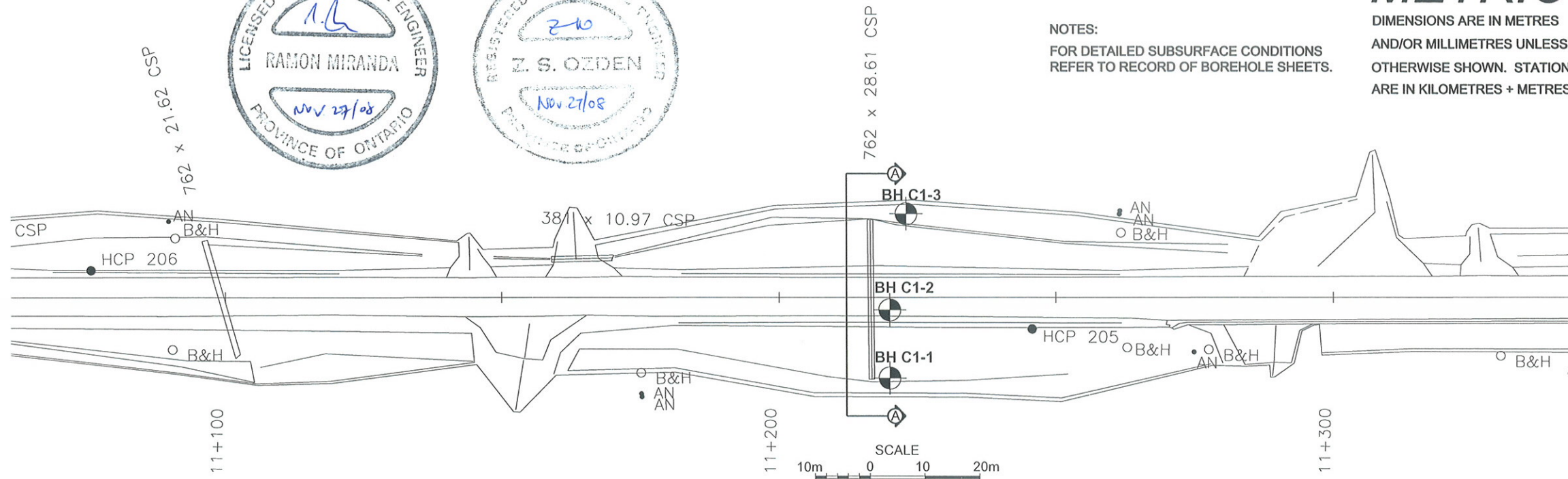
CONT No.
WP: 480-98-00

Highway 522 Port Loring
BOREHOLE LOCATION PLAN &
STRATIGRAPHY (Culvert 5 @ STA. 11+217)

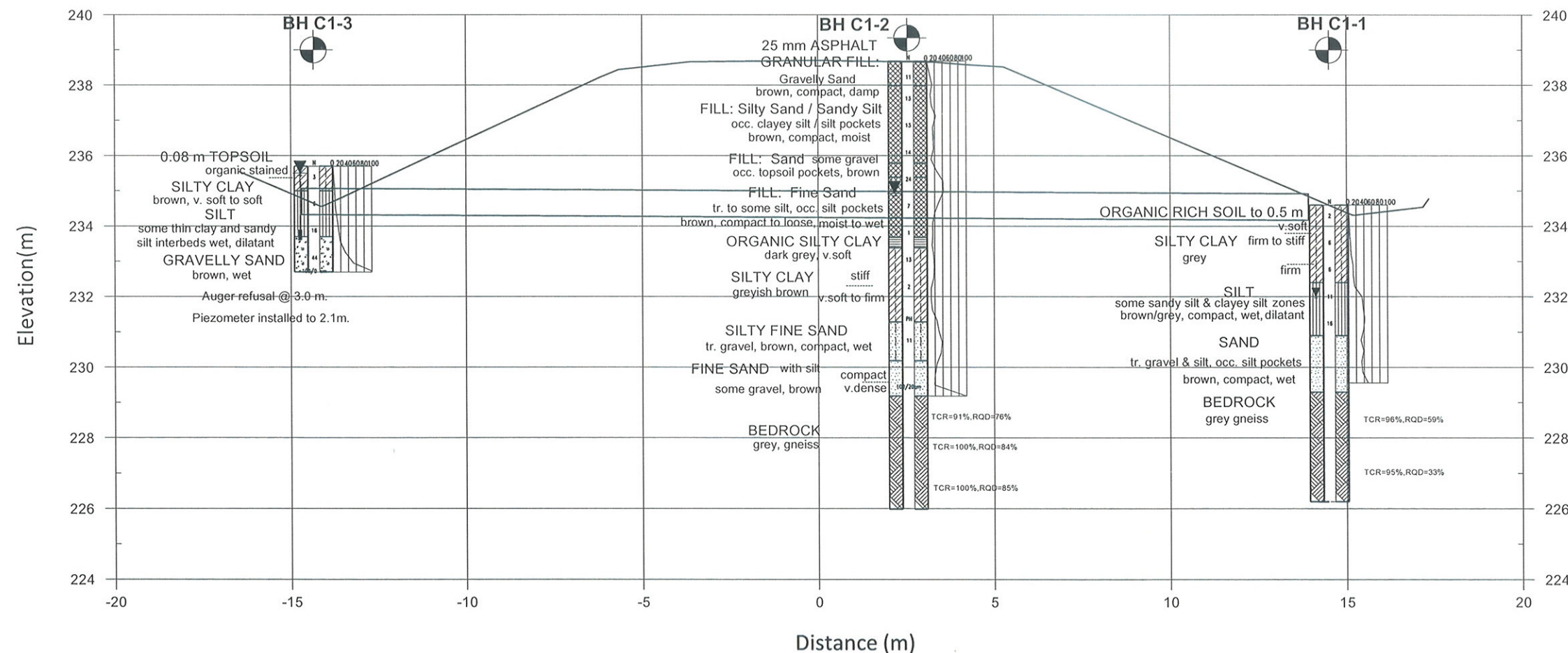
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KEY PLAN
N.T.S.



BOREHOLE LOCATION PLAN



CROSS SECTION (A-A)

LEGEND

- Borehole & Cone
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No.	ELEV.	STATION NO.	OFFSET
C1-1	234.6 m	11+220	14.5 m Rt C/L
C1-2	238.7 m	11+220	2.0 m Rt C/L
C1-3	235.7 m	11+223	15.0 m Lt C/L

NOTE
The boundaries between soil strata have been established only at Borehole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REV.	DATE	BY	DESCRIPTION
Geocres No. 31E-279			
SPT 1218A			DIST
SUBM'D	CHECKED	DATE Sept.2008	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 1

Appendix A

Records of Borehole Sheets

SPT1218A : Highway 522 (Port Loring)

RECORD OF BOREHOLE No C1-1

1 OF 1

METRIC

GWP 480-98-00 LOCATION Sta : 11+220 14.5 m Rt C/L of Hwy 522 ORIGINATED BY RK
 DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger&NQ coring COMPILED BY SS
 DATUM Geodetic DATE 6/18/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)			
								<div>20 40 60 80 100</div> <div>○ UNCONFINED + FIELD VANE</div> <div>● POCKET PENETR. x LAB VANE</div>			
								<div>PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT</div> <div>W_p W W_L</div> <div>WATER CONTENT (%)</div> <div>10 20 30</div>			
234.6	GROUND SURFACE										
0.0	Organic Rich Soil to 0.5 m		1	SS	2	Ψ^*	234				
	dark brown, v soft										
	firm to stiff		2	SS	6		234				
	SILTY CLAY										
	grey										
	firm		3	SS	6		233				
232.4											
2.2	SILT		4	SS	11		232				
	some sandy silt & clayey silt zones										
	brown/grey, compact, wet, dilatant		5	SS	16						
230.9											
3.7	SAND		6	SS	17						
	tr gravel & silt, occ. silt pockets										
	brown, compact, wet		7	SS	18						
229.3											
5.3	BEDROCK		8	RC	TCR=96% RQD=59%		229				
	grey gneiss										
			9	RC	TCR=95% RQD=33%		228				
226.2							227				
8.4	End of borehole										
	water level in open hole @ 2.6 m upon completion (not stabilized)*										
	Dynamic Cone Penetration Test (DCPT) performed adjacent to the borehole from 0 to 5.1 m.										

+ 3 . X 3 : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

coffey geotechnics
SAFETY IS ALWAYS THE FIRST

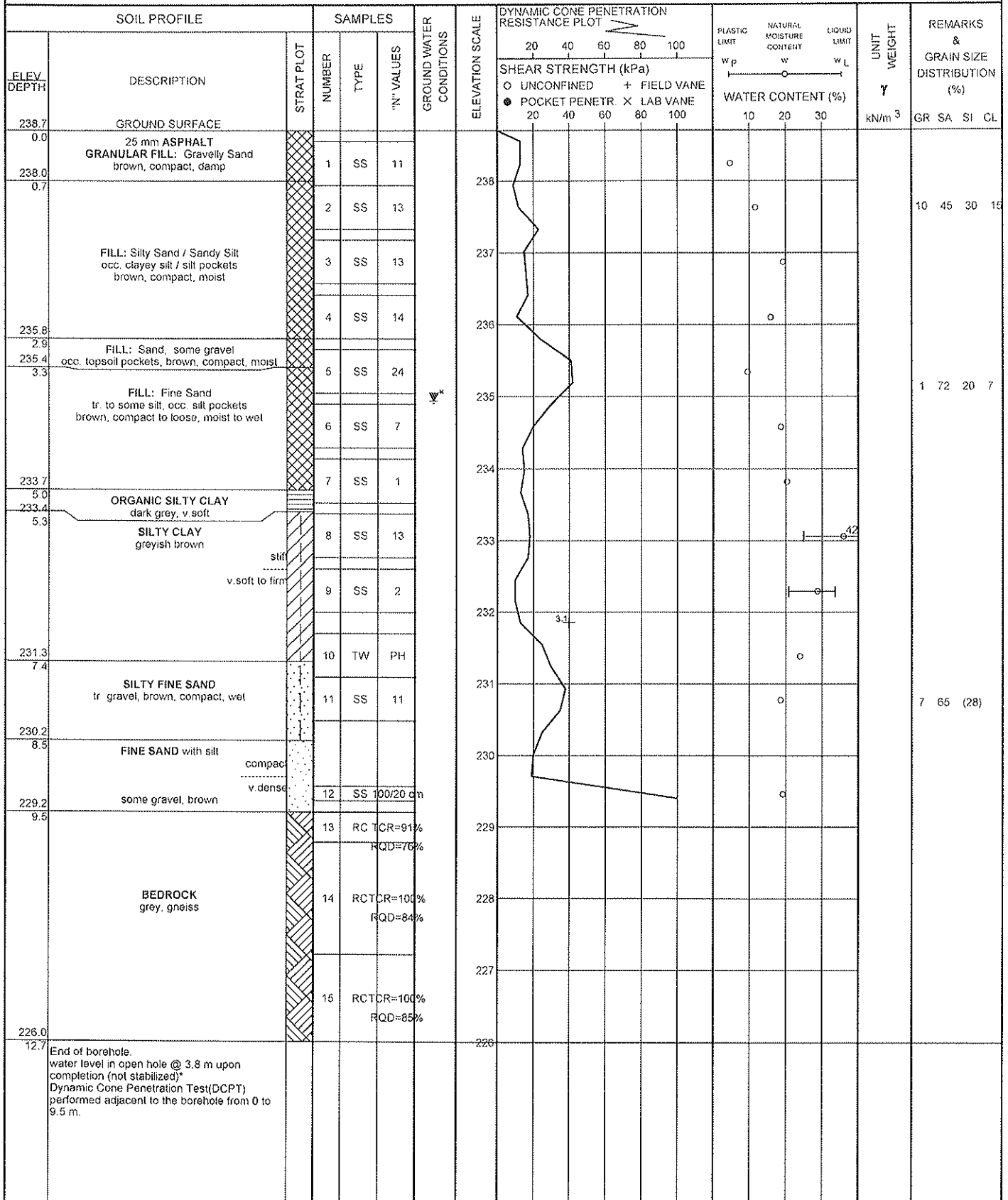
SPT1218A : Highway 522 (Port Loring)

RECORD OF BOREHOLE No C1-2

1 OF 1

METRIC

GWP 480-98-00 LOCATION Sta 11+220 2.0 m RI C/L of Hwy 522 ORIGINATED BY RK
DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger & NQ coring COMPILED BY SS
DATUM Geodetic DATE 6/18/2008 CHECKED BY ZO



+ 3 X 3

Numbers refer to
Sensitivity

20
15 5
10

(%) STRAIN AT FAILURE

coffey geotechnics

SPECIALISTS IN SOILS & ROCKS

SPT1218A : Highway 522 (Port Loring)

RECORD OF BOREHOLE No C1-3

1 OF 1

METRIC

GWP 480-98-00 LOCATION Sta : 11+223 15.0 m Lt C/L of Hwy 522 ORIGINATED BY SK
DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 6/24/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							
235.7 0.0	GROUND SURFACE 0.08 m TOPSOIL		1	SS	3		20	40	60	80	100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	GR SA SI CL
235.0 0.7	SILTY CLAY brown, v. soft to soft organic stained		2	SS	6		WATER CONTENT (%)			10	20	30			
233.7 2.0	SILT some thin clay and sandy silt interbeds wet, dilatant	loose compact	3	SS	16		○ UNCONFINED + FIELD VANE ● POCKET PENETR X LAB VANE								
		compact dense	4	SS	44										
232.7 3.0	GRAVELLY SAND brown, wet		5	SS	100/0										
<p>End of borehole Auger refusal @ 3.0 m Water level in open hole @ 0.15 m upon completion (not stabilized)* Piezometer installed to 2.1 m. Piezometer reading - June 24, 2008 0.15 m - June 25, 2008 0.2 m - July 02, 2008 0.2 m Dynamic Cone Penetration Test (DCPT) performed adjacent to the borehole from 0 to 3.0 m</p>												29 59 (12)	SS5-no sample recovery		

+ ³ × ³ : Numbers refer to
Sensitivity

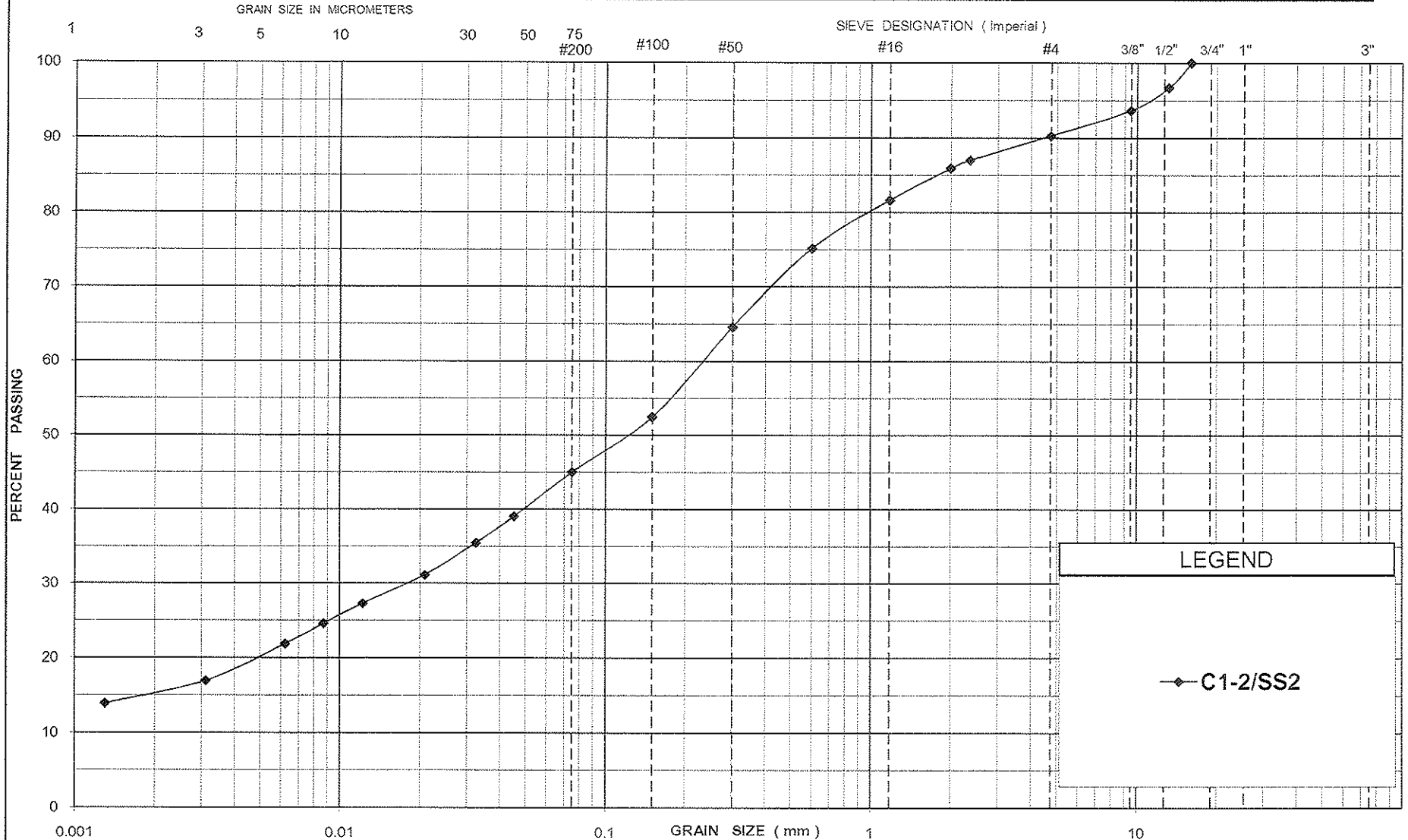
20
15 5
10 (%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL	
			Fine	Medium	Coarse	Fine	Coarse



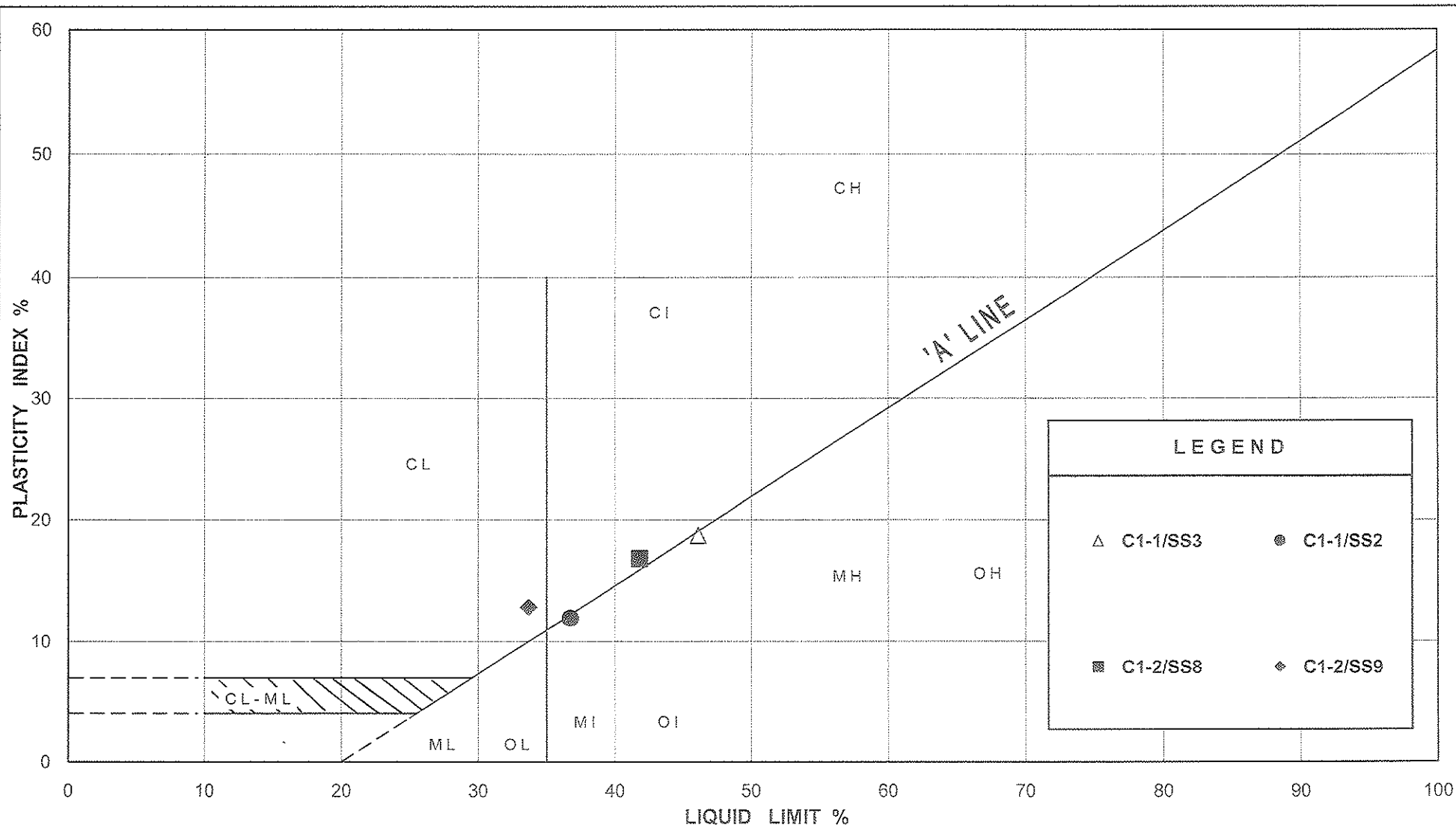
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GRAIN SIZE DISTRIBUTION
FILL: Silty Sand / Sandy Silt, tr. gravel, occ. clayey silt

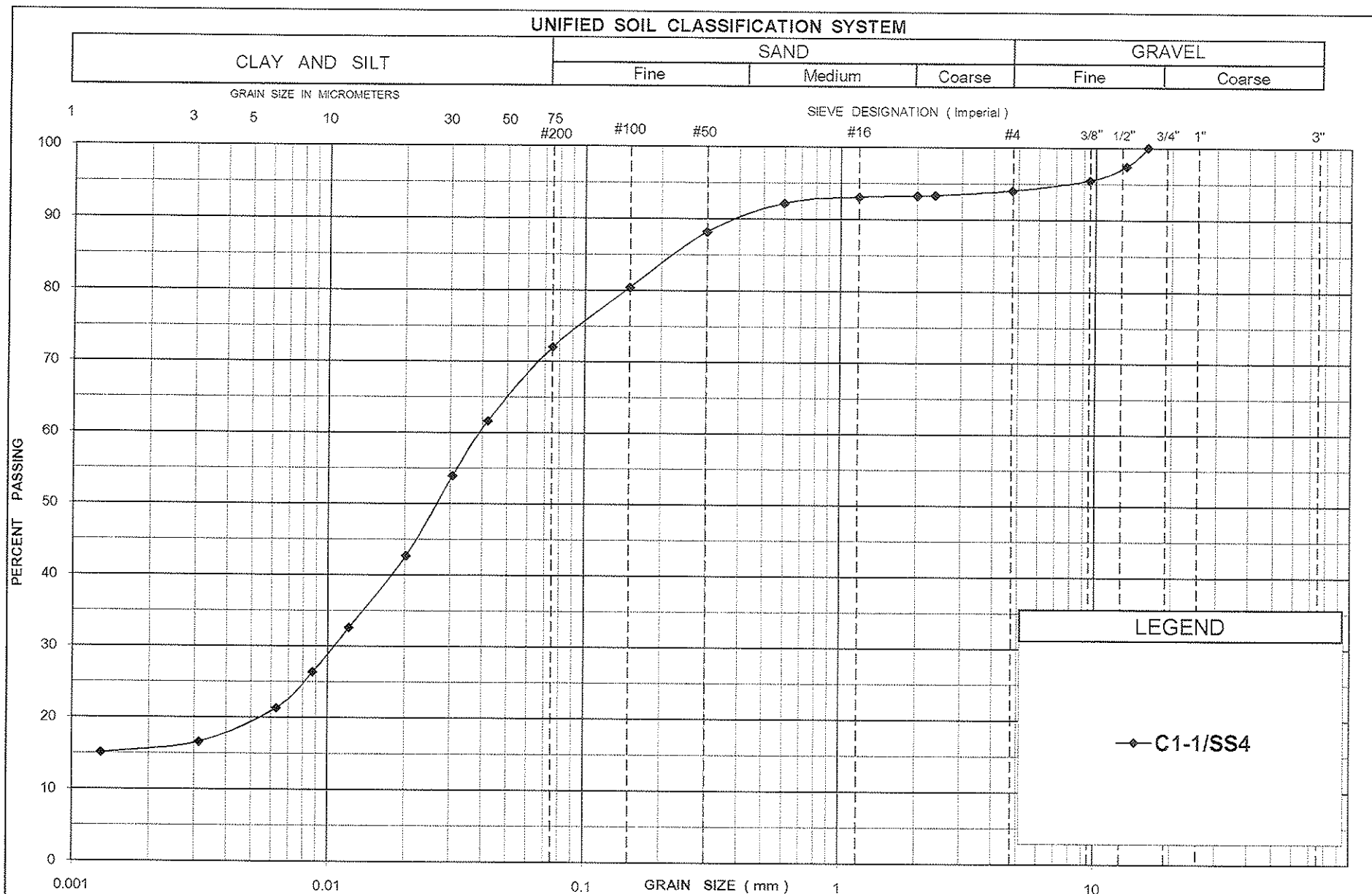
FIGURE No. B-1

REF. No. SPT 1218A

DATE AUGUST 2008



SHAHEEN & PEAKER A Division of Coffey Geotechnics, Inc.	PLASTICITY CHART SILTY CLAY		FIGURE No. B-3
			REF. No. SPT 1218A
			DATE SEPTEMBER 2008



SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

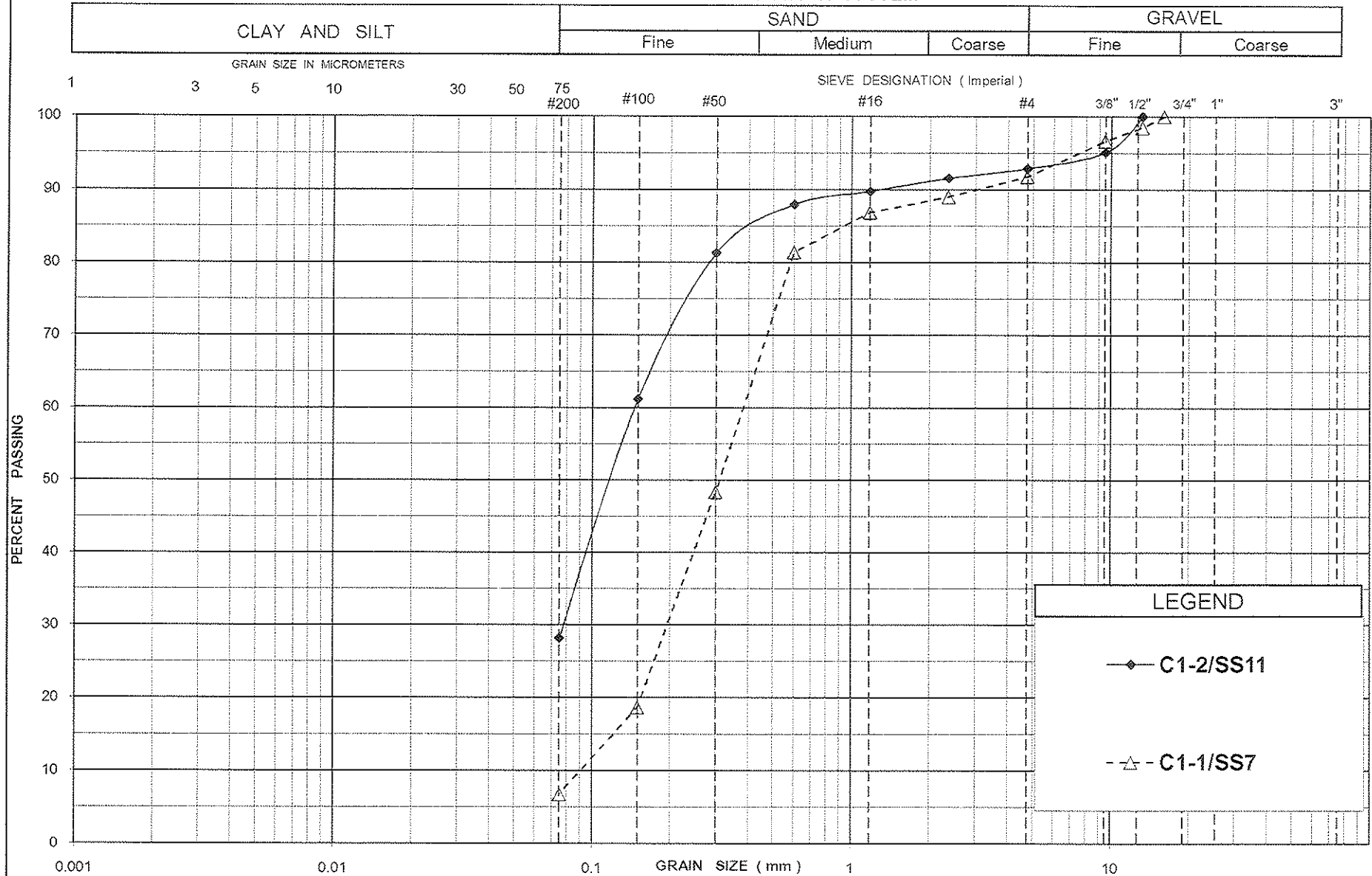
GRAIN SIZE DISTRIBUTION
SILT, some sandy silt & clayey silt zones

FIGURE No. B-4

REF. No. SPT 1218A

DATE AUGUST 2008

UNIFIED SOIL CLASSIFICATION SYSTEM



SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

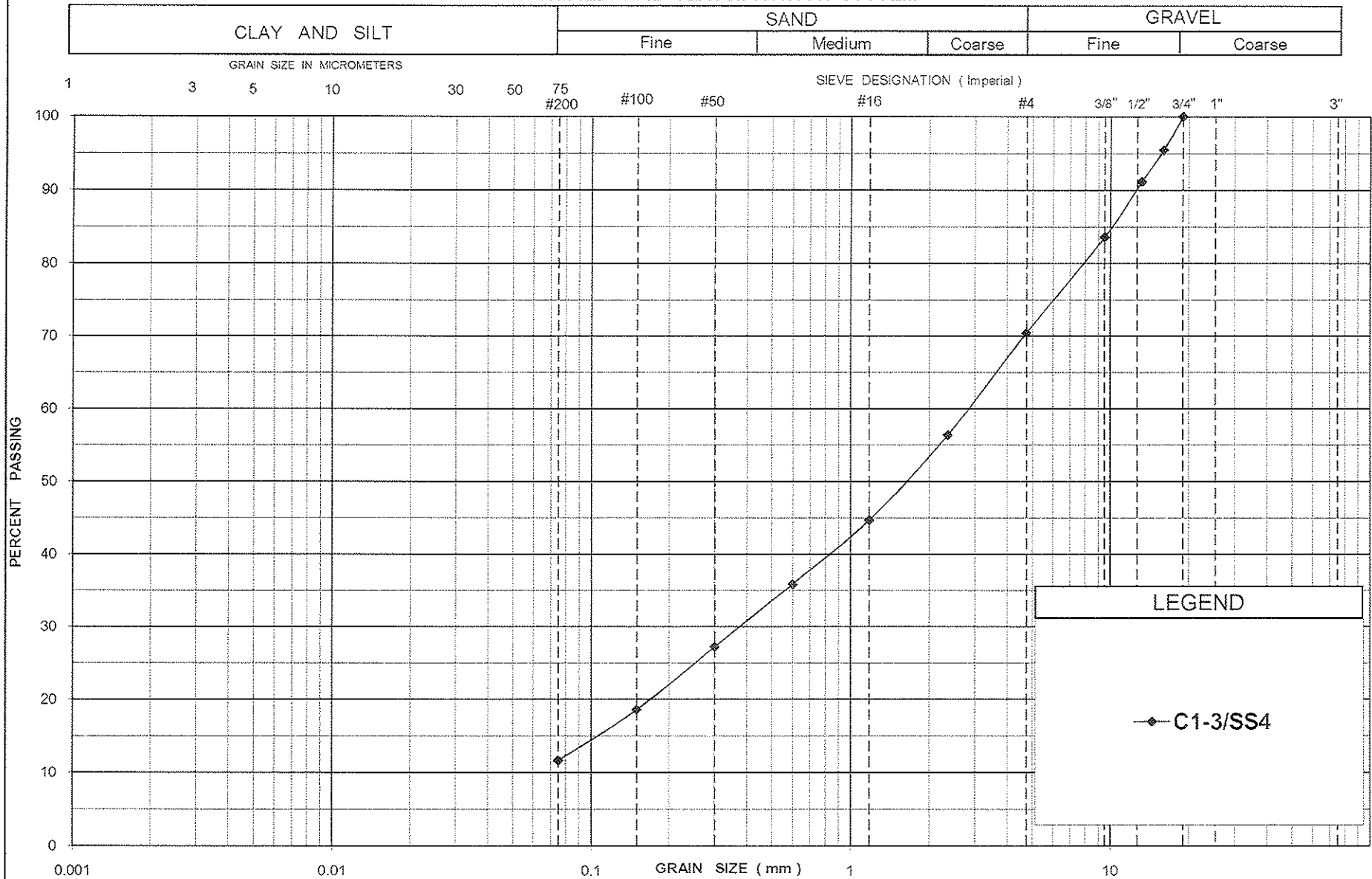
GRAIN SIZE DISTRIBUTION
SILTY FINE SAND to SAND, tr. gravel & silt

FIGURE No. B-5

REF. No. SPT 1218A

DATE AUGUST 2008

UNIFIED SOIL CLASSIFICATION SYSTEM



SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

GRAIN SIZE DISTRIBUTION GRAVELLY SAND

FIGURE No. B-6

REF. No. SPT 1218

DATE AUGUST 2008

Appendix C

Site Photographs



Photograph1. Right (west) end of culvert



Photograph2. Right side of embankment at culvert, looking south



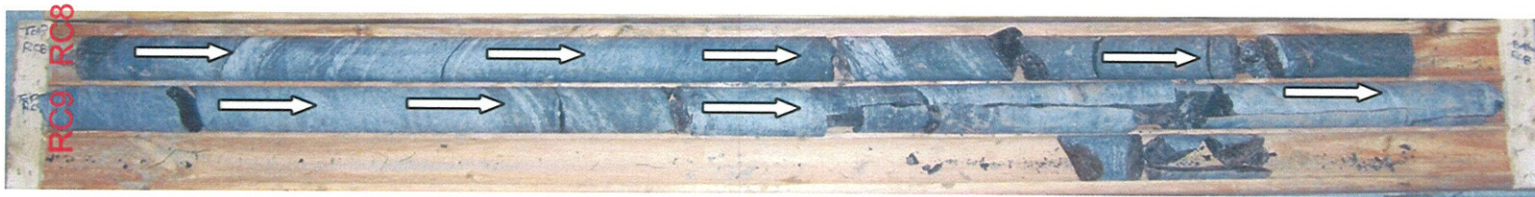
Photograph3. Left side of embankment at culvert, looking south



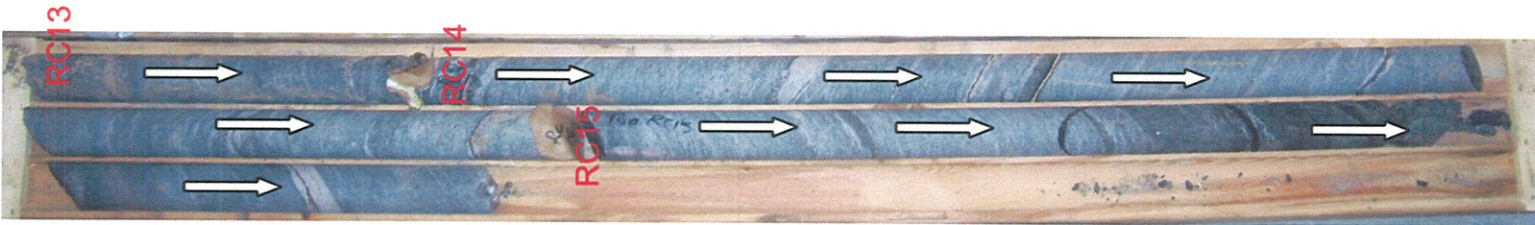
Photograph4. Left (east) end of culvert

Appendix D

Rock Core Photographs



Photograph 1. BH C1-1



Photograph 2. BH C1-2

Appendix E

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCUTRAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINT AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
j_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
P_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
j_w	kN/m ³	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
j	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
j_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(W_L - W_p) / I_p$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W - W_p) / I_p$	i	1	HYDAULIC GRADIENT
j_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(W_L - W) / 1_p$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m ³	DENSITY OF SUBMERED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m ³	SEEPAGE FORCE
j'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT
PROPOSED CULVERT REPLACEMENT (C5) AT
STATION 11+217, HIGHWAY 522 REHABILITATION,
FROM 32.2 KM WEST OF HIGHWAY 524
EASTERLY 6 KM
G.W.P. 480-98-00, DISTRICT 54, SUDBURY
GEOCRES NO. 31E-279**

Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

**SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.**

**Project: SPT1218A
November 27, 2008**



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APPENDICES

APPENDIX F: OPSD

APPENDIX G: LIMITATIONS OF REPORT

**FOUNDATION DESIGN REPORT
PROPOSED CULVERT REPLACEMENT (C5) AT
STATION 11+217, HIGHWAY 522, REHABILITATION, FROM 32.2 KM
WEST OF HIGHWAY 524 EASTERLY 6 KM
G.W.P. 480-98-00, DISTRICT 54, SUDBURY**

5. DISCUSSION AND RECOMMENDATIONS

5.1 UNDERSTANDING OF THE PROJECT

We understand that the existing 28.6 m long, 762 mm diameter CSP culvert at Station 11+217 will be replaced with a new culvert of similar length and similar diameter. The invert elevation of the new culvert will match that of the existing culvert (i.e. El. 234.4 m on the upstream (east) side and El. 234.2 m on the downstream (west) side).

The road grade may be raised by up to about 100 mm due to pavement rehabilitation. There will be no widening of the embankment. During construction, the highway at the culvert location will be closed to traffic.

Three boreholes were drilled at the site, namely Boreholes C1-1 and C1-3 on the west and the east ends (near the toe of the embankment), while the third borehole (Borehole C1-2) was put down from the top of the embankment, on the paved shoulder of the existing highway, immediately adjacent to the existing culvert.

Below some 5.0 m embankment fill and an underlying 0.3 m thick layer of organic silty clay in Borehole C1-2, and some surficial topsoil and organic rich soils in Boreholes C1-1 and C1-3, all three boreholes show the presence of a silty clay deposit. The surface of the inorganic silty clay was contacted at El. 234.1, 233.4 and 235.3 m in Boreholes C1-1, C1-2 and C1-3, respectively. Boreholes C1-1 and C1-3 encountered silt beneath the silty clay at depths of 2.2 m and 0.7 m or at El. 232.4 m and 235.0 m, respectively. This silt deposit is 1.5 and 1.3 m thick, respectively and is underlain by sand (Borehole C1-1) and gravelly sand (Borehole C1-3) at depths of 3.7 to 2.0 m. Borehole C1-3 was terminated in the gravelly sand upon encountering refusal at 3.0 m (El. 232.7 m). In Borehole C1-1, the sand deposit was found to extend to the surface of the bedrock at 5.3 m/El. 229.3 m.

In Borehole C1-2 (the central borehole), a granular soil deposit consisting of silty fine sand and fine sand was contacted underlying the silty clay. The granular deposit in Borehole C1-2 extends to a depth/elevation of 9.5/229.2 m and is further underlain by bedrock.

The groundwater level at the time of our investigation was found at about ground surface (o.g.). The water level at the site could be subject to fluctuations due to major weather events and seasonal variations.

5.2 CULVERT FOUNDATION SUPPORT

As was mentioned before, the invert elevations for the culvert are 234.4 m at the upstream side and 234.2 m on the downstream side. With a minimum bedding thickness of 0.25 m under the pipe, on the downstream side the excavation will be carried down to El. 234.0 ±m. The borehole drilled on the west side (Borehole C1-1) near the proposed pipe outlet area shows the presence of a firm to stiff (i.e. suitable) silty clay at or below El. 233.8 m. This means that a minor over-excavation and replacement with suitable granular soil may be required.

Borehole C1-3, drilled near the inlet area, shows the presence of a silt deposit at the anticipated excavation depth to below the bedding for the pipe (i.e. to El. 234.2 m). At this elevation the undisturbed and properly dewatered silt would be suitable to support the pipe. It must, however, be pointed out that the silt is a dilatant material which must be stabilized by proper dewatering otherwise excessive settlements may ensue after the road excavation is backfilled, as will further be elaborated in Section 5.5 of this report.

In the borehole put down more centrally across the roadway (i.e. Borehole C1-2), the surface of the suitable soil was contacted at El. 233.4 m. This means that some engineered fill may need to be placed after excavating the organic silty clay to the surface of the inorganic, suitable silty clay, unless all the unsuitable soils have been replaced under the existing pipe and have been replaced with proper granular engineered bedding fill (Borehole C1-2 was not drilled directly on top of the existing culvert, but adjacent to it). This aspect can be clarified during construction by digging shallow test pits after removing the existing pipe.

Since sub-excavation will likely be required and the underlying subgrade soils are only marginally suitable, as well due to the presence of disturbance prone to silts near the outlet, for this project the use of a CSP culvert (similar to the existing culvert) would, in our opinion be a better choice in comparison with more rigid concrete structures, since a CSP culvert would be more flexible and therefore less sensitive to total and differential settlements. As well, to our knowledge their installation time would typically be shorter. This could be an important factor in the selection of culvert type, since according to our understanding, the highway will be closed during the construction and the use of CSP type culvert may expedite the work.

Provided that all the unsuitable soils are removed and where necessary replaced with suitable granular soils (where grade needs to be raised after sub-excavation, e.g. probably at Borehole C1-2 and possibly at Borehole C1-1 location), there should be no problems with

bearing resistance and settlements, since there will virtually no load increases over and above existing conditions (i.e. no widening and only up to 100 mm grade raise of the road). However, for completeness the following geotechnical resistances can be assumed for undisturbed subgrade soils.

Bearing Resistance at U.L.S.	=	120 kPa
Factored Geotechnical Resistance at S.L.S.	=	50 kPa

Under the embankment, the value at SLS is less than the existing embankment loading. This however is not considered to be a problem since the overburden under the existing embankment would have fully consolidated/settled under the existing embankment loads. Therefore, since there will be little or no additional loading, there should be negligible additional settlements. However, a settlement of about 25 mm should be allowed for, due to slight increased load for pavement rehabilitation and soil exchange as well as for rebound during construction (i.e. the embankment will be excavated) and re-settlement after backfilling. Based on this, it is our opinion that cambering is not required.

5.3 BEDDING

The bedding material should be placed as soon as practicable after the preparation of the subgrade, as discussed, its inspection and approval. The bedding should be in accordance with the appropriate standards (e.g. OPSD-802.010 and 802.014) for flexible pipes or OPSD-802.030, 031, 032 or 034 (for rigid pipes) and should consist of not less than 250 mm thick layer (after compaction) of approved granular material, such as Granular 'B' Type II or Granular 'A.' (Granular 'B' Type II is preferred under the pipe.) Under the pipe, the thickness of the bedding material may need to be increased depending on the site conditions at the time of construction. The bedding material should be compacted to at least 95% of the material's SPMDD using a suitably light compactor to ensure that the underlying subgrade is undisturbed. If the bedding is to consist of a poorly graded material such as clear crushed stone, a suitable geotextile should be placed as a separator at the bottom and sides of the excavation, as well as the top. However, the use of poorly graded bedding is not recommended for this project.

5.4 BACKFILLING

The bedding and embedment material should be extended along the sides to cover the pipe. The selection and placing of the backfill should be in accordance with OPSD-802.010 and OPSD-802.014 for flexible pipes or appropriate standards for rigid pipes. The backfill should consist of free-draining, non-frost susceptible granular materials such as Granular 'A' or 'B' (OPSS-1010). All granular backfill materials should be placed in thin lifts (i.e. not exceeding 300 mm before compaction) and should be compacted to at least 96% of the material's SPMDD. The Granular 'A' base and the Granular 'B' sub-base courses should be compacted to 100% of the SPMDD. The fill should be placed simultaneously on each side

of the pipe to prevent lateral dislocation of the pipe. Uplift of the pipe must be prevented by means of dewatering and/or placing sufficient fill above it.

We would like to point out that the performance of flexible pipe culverts is largely dependent on the side support provided by the backfill and the adjacent soils. The use of proper backfill material and especially good compaction are, therefore, necessary for proper side support. The use of heavy compaction equipment should, however, be avoided immediately adjacent and above the pipes, as per MTO practice. During backfill placement, the height of the backfill should be maintained at approximately same level on both sides of the pipe, to avoid lateral displacement of the pipe.

Proper frost treatment is required in accordance with OPSD-803.030 or 803.031, whichever is applicable.

The use of vibratory compaction equipment behind the culvert should be restricted in size as per current MTO practice.

5.5 CONSTRUCTION

Based on the information provided by D.M. Wills Associates, Highway 522 at the project site will be completely closed without any detour or roadway protection during the culvert replacement. The construction will be carried out without shoring.

The flow of water in the existing watercourse will need to be maintained during the construction. This can be achieved by placing a temporary pipe for the construction period or using the existing culvert for this purpose until the new culvert is built.

Depending on the groundwater level encountered at the time of the construction, some form of dewatering will likely be required to facilitate the construction and to preserve the load carrying capability of the founding soils. Based on the borehole data, dewatering will likely be more critical at Borehole C1-3 location (i.e. on the east side), where a dilatent silt deposit was contacted at the proposed invert elevation. In general, the groundwater, where necessary, can be depressed by means of closely spaced and strategically placed filtered sumps. On the west side, it may also be necessary to depressurize the water bearing gravelly sand underlying the silt below El. 233.7 m. However, in this manner the groundwater will probably be depressed by not much more than about 0.6 m. To depress the groundwater level further deeper, other methods such as deep wells and/or well points would be required. In this instance, the well points would need to be extended into the granular soils underlying the silty clay (Boreholes C1-1 and C1-2) and the silt (Borehole C1-3). The presence of the bedrock (Boreholes C1-1 and C1-2) and the auger refusal depth should be taken into consideration when designing this type of dewatering system. It is however unlikely that such a system will be required.

We recommend that the contractor be made aware of possible dewatering requirements to facilitate the construction. In this respect, the contractor may choose to dig some test pits to investigate conditions at the time of construction and the necessity for dewatering, and the methods that may be required for this purpose.

As mentioned before, the silt is a dilatant material which, especially in the presence of water, can easily dilate, a condition which can be recognized by the liverish, jelly-like appearance of the soil. If the pipe is placed on disturbed, dilated soil, excessive settlements can occur after backfilling. For these reasons, we recommend that if at all possible, the construction be carried out during a dry period. As well, care should be taken to avoid disturbing subgrade soils by minimizing construction traffic (including foot traffic) and minimizing vibrations. As well, stripping should be carried out under geotechnical supervision to acceptable subgrade level and the bedding material and/or soil to raise the grade should be placed immediately after exposing the suitable subgrade, its inspection and approval. We recommend that the material placed above the approved subgrade to raise the grade and/or as a bedding consist of Granular 'B' Type II or Granular 'A' material. Where the subgrade is relatively weak, we recommend that the Granular 'B' Type II or Granular 'A' material be pushed into the inorganic subgrade, if necessary, in order to improve the subgrade to make it firmer. As well, where subgrade is relatively weak, the first lift of backfill may need to be up to 0.8 m thick.

The contractor should also be made aware of the possible presence of cobbles and boulders in the embankment fill and in the underlying overburden.

We recommend that the contractor be alerted by means of an NSSP that special care is needed to avoid disturbing the founding soils. As well, the contractor should be required to submit their dewatering and excavation proposal to the CA for information purposes.

The construction of the culvert should be in accordance with OPSS 421.

All excavations should be carried out in accordance with the Province's Occupational Health and Safety Act (OHSA), O. Reg. 213/91, as well as the following:

- SP 105 S19 – Protection Systems
- SP 902 S01 – Excavation and Backfilling - Structures

In accordance with the Province's Safety Regulation, the following soil classification would be applicable.

Granular Pavement Fill	Type 3 soil
Embankment Fill	Type 3 soil above water level
	Type 4 soil below water level
Topsoil and Organic Silt	Type 4 soil below water level
Silt	Type 4 soil
Silty Clay	Type 3 soil above water level
	Type 4 soil below water level

Regardless of the classification given above, we recommend that side slopes above water level for temporary excavations with unsupported side slopes should be no steeper than 2H:1V. This can be steepened, if approved by the QEV, but no steeper than 1 1/2H:1V. Below water level (e.g. if the site was not properly dewatered), flatter side slopes would be required.

5.6 EROSION PROTECTION

Erosion and scour protection should be provided at the culvert inlet and outlet (including the side slopes). The erosion/scour protection should be designed by a specialist River Engineer/Scientist (as erosion and scour largely depend on the velocity of water in the watercourse and its regime) who is familiar with the findings of this report. The following are some general suggestions.

We recommend that a cut-off (apron) wall be constructed both at the inlet and the outlet to prevent seepage beneath and around the culvert, especially through the granular bedding and granular backfill around the culvert. Beneath the culvert, the cut-off wall should extend to a suitable depth (i.e. below any possible scour depth).

Based on the available borehole data, the soil at both the inlet and outlet will likely consist of silty clay which is not highly erodible but the underlying silt contacted in Borehole C1-1 (at the outlet) is a highly erodible material. At the inlet area (i.e. Borehole C1-3) the highly erodible silt is at the invert level (i.e. the overlying silty clay in this borehole is very thin). Erosion protection measures should take this aspect into consideration.

5.7 BEARING SURFACES

We recommend that all bearing surfaces should be inspected and approved by a qualified Geotechnical Engineer (QVE).

5.8 FROST PROTECTION

Design frost protection for the project area is 1.8 m. Therefore, a permanent soil cover of 1.8 m or its thermal equivalent of artificial insulation is required for frost protection of foundations. In case of riprap (rock fill), only one-half of the rock fill thickness should be assumed to be effective in providing frost protection.

6. CLOSURE

We recommend that once the details of the culvert are finalized, our recommendations be reviewed for their specific applicability. The Limitations of Report, as quoted in Appendix F, are an integral part of this report.

SHAHEEN & PEAKER



Gwangha Roh, Ph.D.



Ramon Miranda, P. Eng.



Zuhtu S. Ozden, P. Eng.

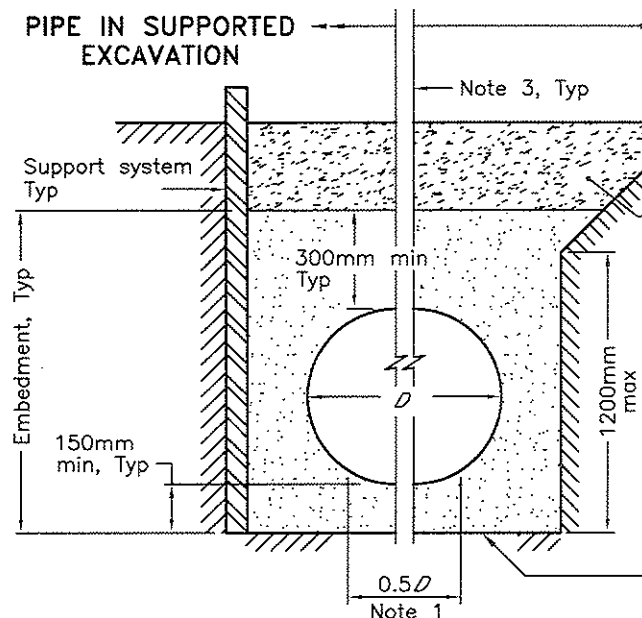


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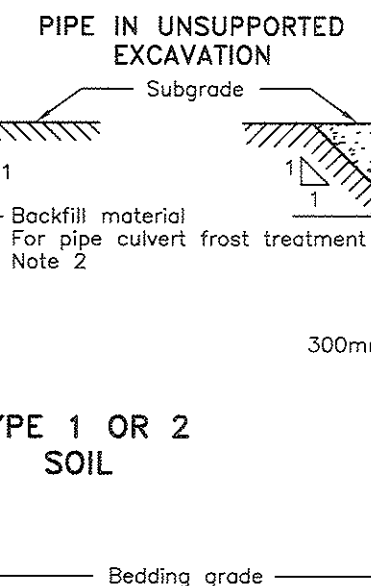
Appendix F

OPSD

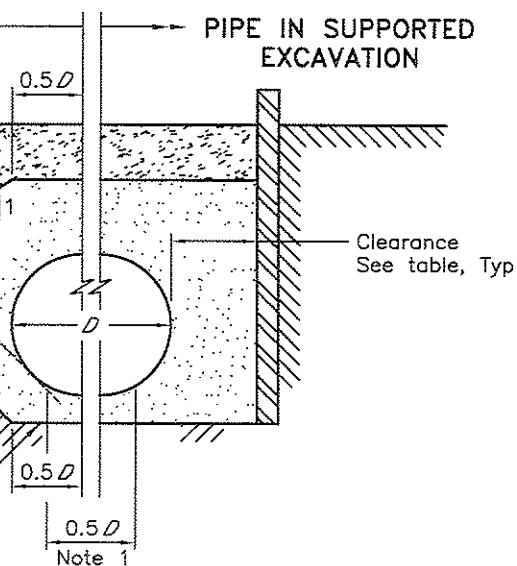
PIPE IN SUPPORTED EXCAVATION



PIPE IN UNSUPPORTED EXCAVATION



PIPE IN SUPPORTED EXCAVATION



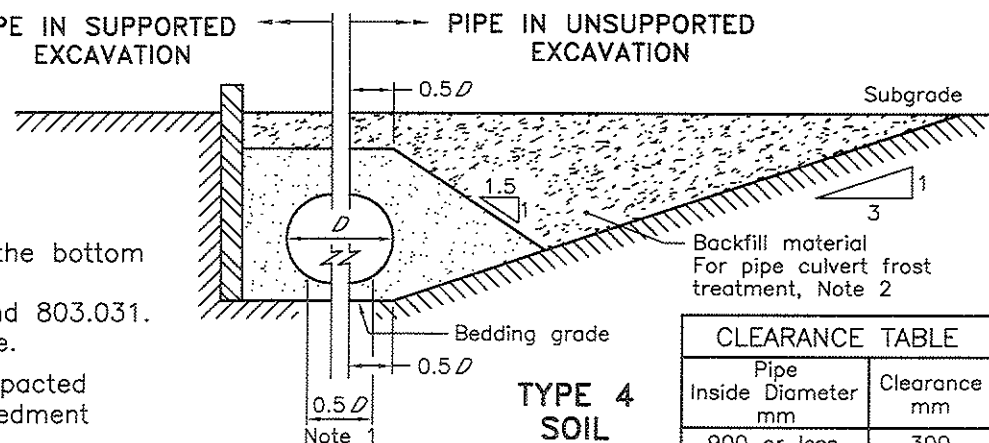
LEGEND:

D - Inside diameter

NOTES:

- 1 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
 - 2 Pipe culvert frost treatment according to OPSD-803.030 and 803.031.
 - 3 Condition of trench is symmetrical about centreline of pipe.
- A Granular material placed in the haunch area shall be compacted prior to placing and compacting the remainder of the embedment material.
- B Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
- C All dimensions are in metres unless otherwise shown.

PIPE IN SUPPORTED EXCAVATION



TYPE 4 SOIL

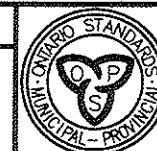
CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

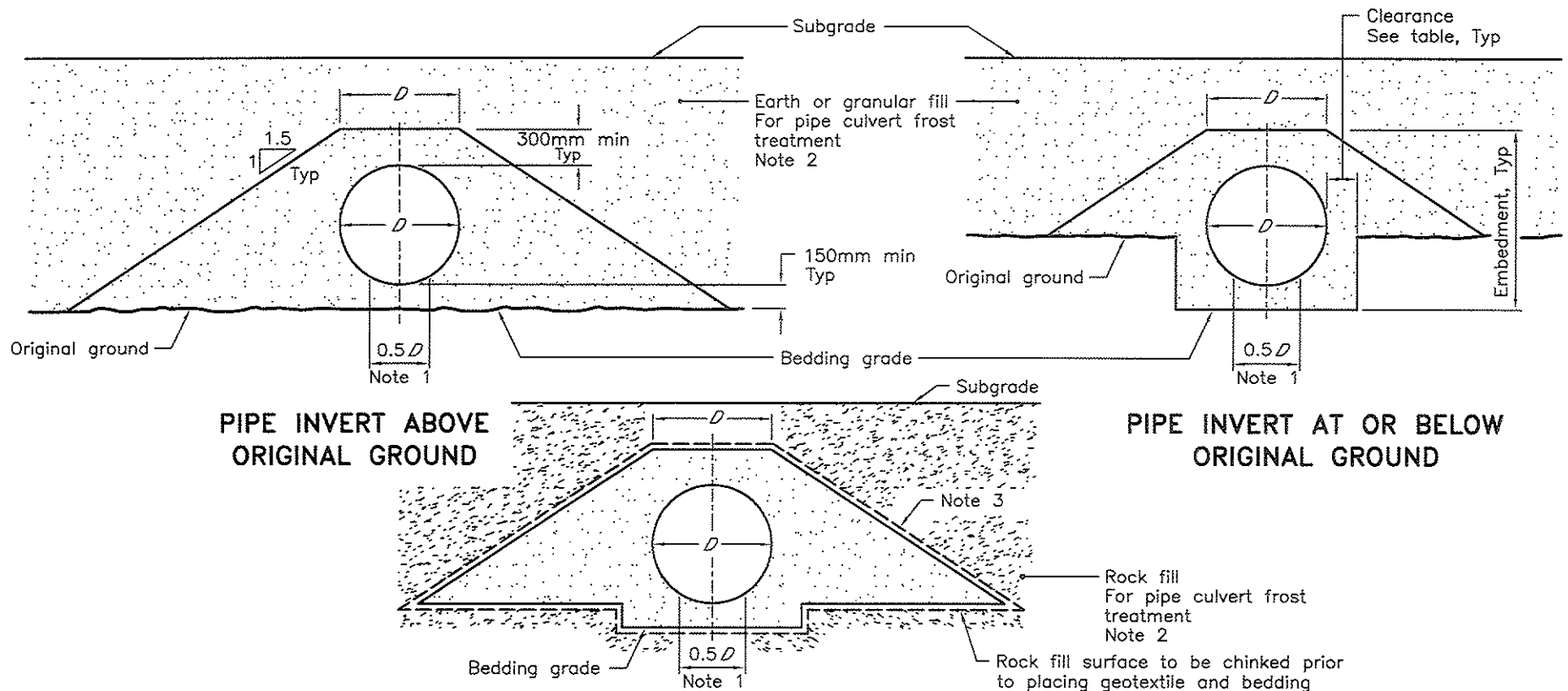
Nov 2005

Rev 1

FLEXIBLE PIPE
EMBEDMENT AND BACKFILL
EARTH EXCAVATION



OPSD - 802.010



**PIPE INVERT ABOVE
ORIGINAL GROUND**

**PIPE INVERT AT OR BELOW
ORIGINAL GROUND**

**PIPE EMBEDMENT
WITH ROCK FILL UNDER AND OVER THE PIPE**

LEGEND:

D - Inside diameter

NOTES:

- 1 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
 - 2 Pipe culvert frost treatment according to OPSD-803.030 and 803.031.
 - 3 Embedment material to be wrapped in non-woven geotextile when specified.
- A Granular material placed in the haunch area shall be compacted prior to placing and compacting the remainder of the embedment material.
- B All dimensions are in metres unless otherwise shown.

CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

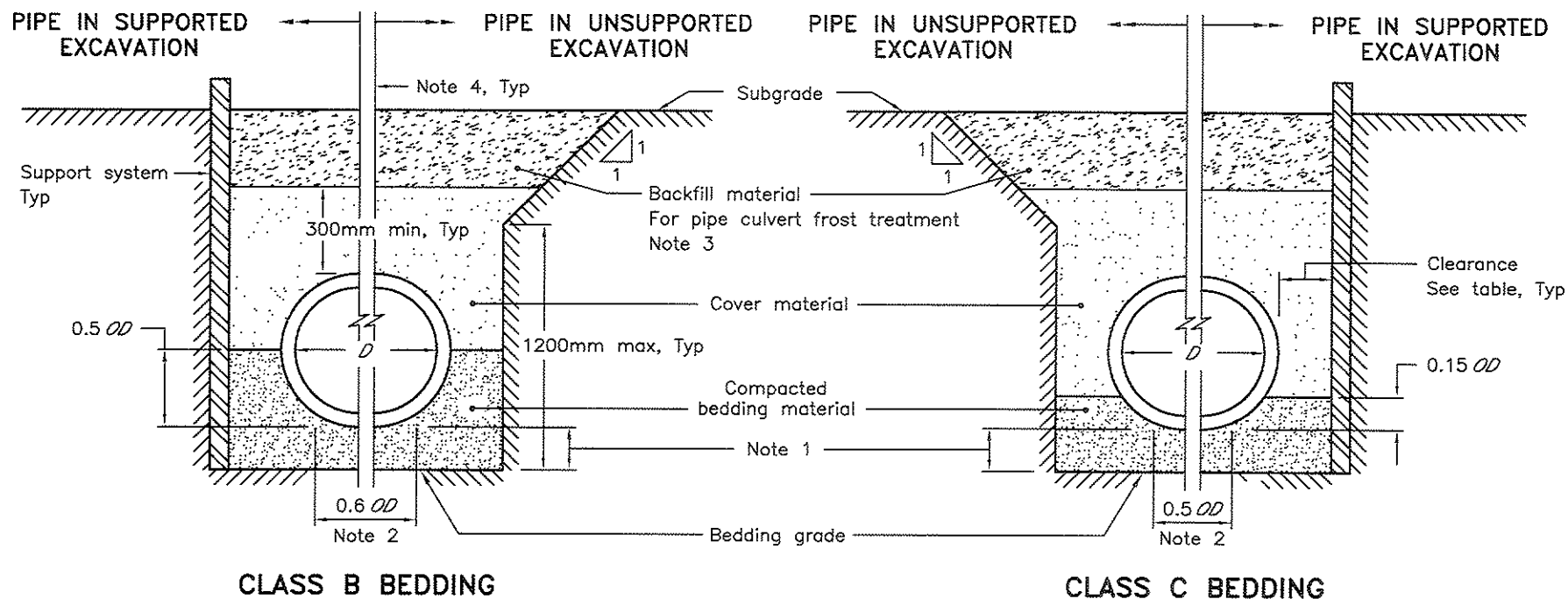
Nov 2005

Rev 1

**FLEXIBLE PIPE EMBEDMENT
IN EMBANKMENT
ORIGINAL GROUND: EARTH OR ROCK**

OPSD - 802.014





NOTES:

- 1 The minimum bedding depth below the pipe shall be $0.15D$. In no case shall this dimension be less than 150mm or greater than 300mm.
 - 2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
 - 3 Pipe culvert frost treatment according to OPSD-803.030 and 803.031.
 - 4 Condition of trench is symmetrical about centreline of pipe.
- A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
- B All dimensions are in metres unless otherwise shown.

LEGEND:

D - Inside diameter
 OD - Outside diameter

CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

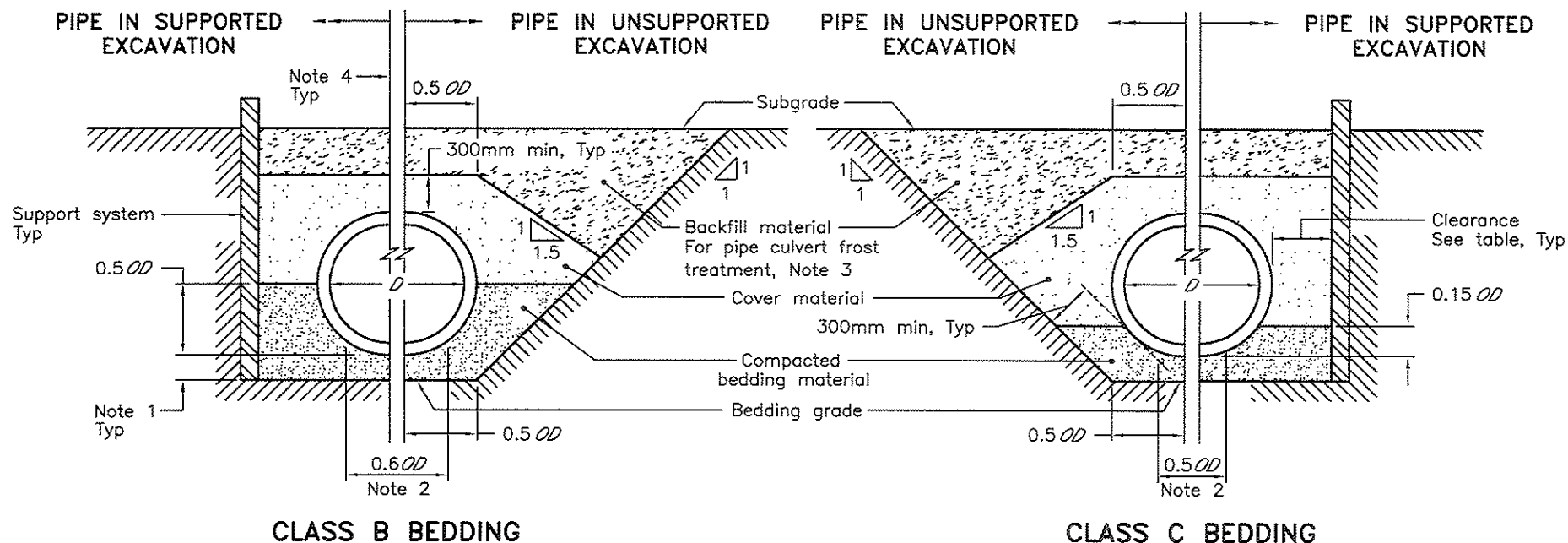
Nov 2005

Rev 1

**RIGID PIPE BEDDING,
 COVER, AND BACKFILL
 TYPE 1 OR 2 SOIL - EARTH EXCAVATION**

OPSD - 802.030





NOTES:

- 1 The minimum bedding depth below the pipe shall be $0.15D$. In no case shall this dimension be less than 150mm or greater than 300mm.
 - 2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
 - 3 Pipe culvert frost treatment according to OPSD-803.030 and 803.031.
 - 4 Condition of trench is symmetrical about centreline of pipe.
- A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
- B All dimensions are in metres unless otherwise shown.

LEGEND:

D - Inside diameter
 OD - Outside diameter

CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

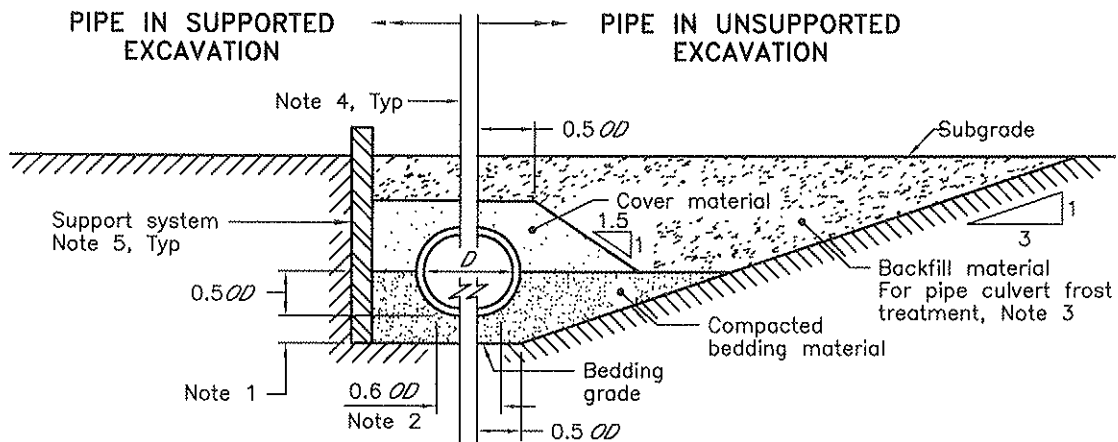
Nov 2005

Rev 1

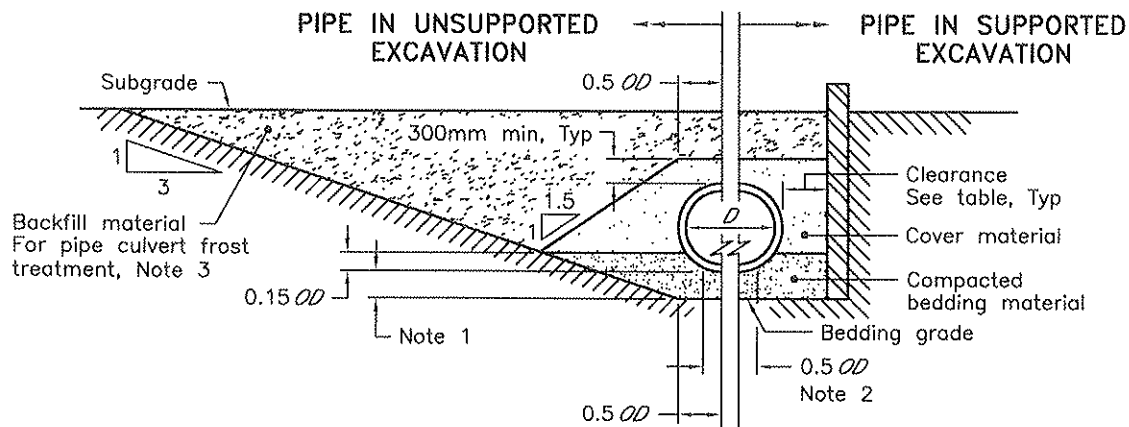
RIGID PIPE BEDDING,
 COVER, AND BACKFILL
 TYPE 3 SOIL - EARTH EXCAVATION

OPSD - 802.031





CLASS B BEDDING



CLASS C BEDDING

LEGEND:

D - Inside diameter
 OD - Outside diameter

CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

NOTES:

- 1 The minimum bedding depth below the pipe shall be $0.15D$. In no case shall this dimension be less than 150mm or greater than 300mm.
 - 2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
 - 3 Pipe culvert frost treatment according to OPSD-803.030 and 803.031.
 - 4 Condition of trench is symmetrical about centreline of pipe.
- A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
- B All dimensions are in metres unless otherwise shown.

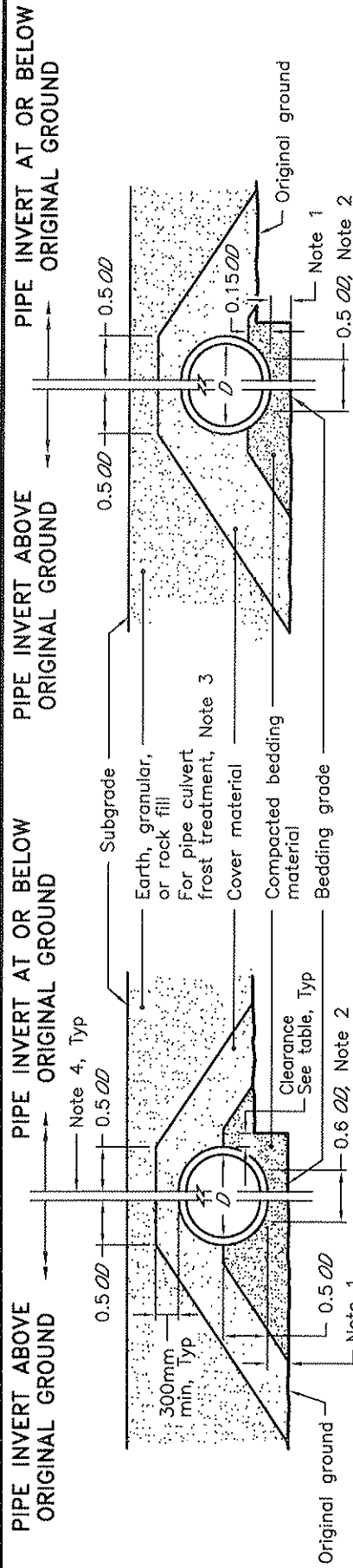
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005 Rev 1

**RIGID PIPE BEDDING,
 COVER, AND BACKFILL
 TYPE 4 SOIL - EARTH EXCAVATION**

OPSD - 802.032

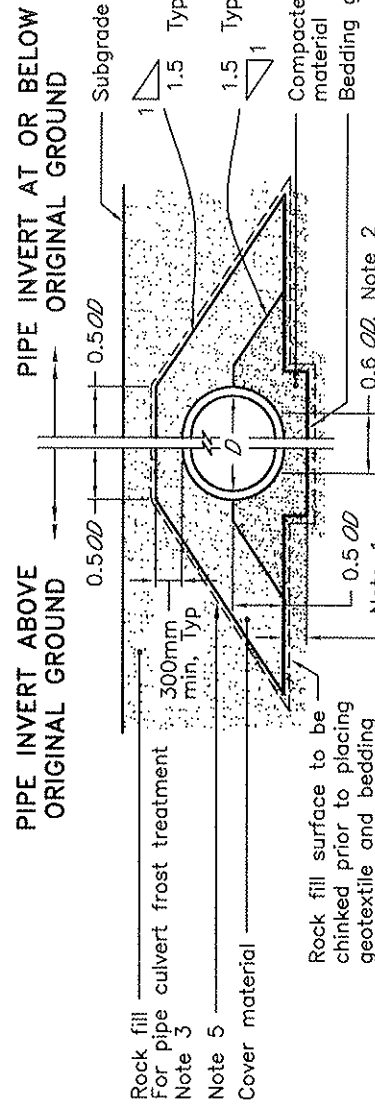




CLASS B BEDDING

EARTH AND ROCK EXCAVATION

CLASS C BEDDING



PIPE BEDDING AND COVER WITH ROCK FILL UNDER AND OVER THE PIPE

CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

NOTES:

- The minimum bedding depth below the pipe shall be $0.15D$, except on a rock foundation where the minimum bedding depth shall be $0.25D$. In no case shall the minimum dimension be less than 150mm or the maximum dimension exceed 300mm.
- The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
- Pipe culvert frost treatment according to OPSP-803.030 and 803.031.
- Condition of trench is symmetrical about centreline of pipe.
- Bedding and cover material to be wrapped in non-woven geotextile when specified.

A All dimensions are in metres unless otherwise shown.

LEGEND:

D - Inside diameter
 OD - Outside diameter



ONTARIO PROVINCIAL STANDARD DRAWING

RIGID PIPE BEDDING AND COVER
 IN EMBANKMENT

Nov 2005	Rev	1

ORIGINAL GROUND: EARTH OR ROCK

OPSD - 802.034

Appendix G

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Shaheen & Peaker, A Division of Coffey Geotechnics Inc. at the time of preparation. Unless otherwise agreed in writing by Shaheen & Peaker, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Shaheen & Peaker accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.